Stress-Strain Analysis of a Concrete Dam in Predominantly Anisotropic Residual Soil

M.F. Leão, M.P. Pacheco, B.R. Danziger

Abstract. Studies of foundations for dams should take into account the study of the behavior of the rocky mass, primarily the stresses and strains expected during the construction and post-construction period. When these structures are supported on sound rock, the use of concrete dams is favored. However, when the foundation is made up of soil or soft rock, earth dams are the most commonly employed technical solution. The study investigates a concrete-gravity dam in the Dominican Republic, with a foundation comprised of alternating horizons of residual soil and weathering soft rock, presenting marked anisotropy and the presence of relict structures. A high incidence of discontinuities gave rise to a more detailed study of the stress-strain behavior of the foundation, taking into account the discontinuity attitudes and the results of direct shear tests carried out parallel to and orthogonal to the discontinuities. By means of 2-D finite element simulations, an anisotropic constitutive model was adopted (the Jointed Rock Model-JRM) capable of modeling up to two preponderant discontinuity directions. The numerical analyses clearly showed the utility of the model in the selection of foundation reinforcement options, in this case through the use of cut-offs to increase the stability conditions.
Keywords: anisotropic constitutive model, anisotropy, concrete-gravity dam, residual soil.

1. Introduction

When investigating a dam site, essential geological and geotechnical aspects must be analyzed, such as: (i) the quality of the materials existing in the abutments and the foundation, (ii) the availability of these rock and soil deposits, and (iii) their relationship to geotechnical parameters of permeability, strength and deformability. Studies of foundations for dams should take into account the behavior of the rock mass, primarily the stresses and strains expected during the construction and post-construction period. When the foundation of the dam is composed of heterogeneous residual soil and weathered rock, there is great concern about the settlement capacity due to the concentration of stresses. The dam may undergo differential settlements depending on the weight transmitted to the foundation and the stratified soil.

These characteristics are difficult to predict when the selected sites present weathering profiles that, in tropical countries, are usually very thick. The presence of transition layers, partially composed of residual soils and rocks in differing degrees of alteration, requires special treatment.

In spite of these peculiarities, several dam projects have been developed on weathering profile, such as Obruk on basalt (Kocbay & Kilic, 2006) and Keban on karst marble and shale (Ertunç, 1999), both in Turkey; Tianhuanping on rhyolite and andesite (Wang & Liu, 2005) in China; Porthimund on charnockites (Ramana & Gogte, 1982), Uri and Nathpa-Jhakri on schists (Behrestaghi *et al.*, 1996) all in India; Hickory Log Creek on mica-schist (Rogers *et al.*, 2006) in the United States; and Clyde on schists (Macfarlane, 2009) in New Zealand. However, it is important to point out that there are still only limited scientific publications on schist materials as foundation, mainly for concrete-gravity dams, which often leads to a preference for the construction of earth or rock dams.

In this context, a case study of the San Juan dam on the Samaná Peninsula, in the Dominican Republic is presented. From a geological-geotechnical point of view, the foundation and abutments are supported on highly anisotropic young residual soil, with marked relict structures, derived from schist and marble, as well as soft rock layers interbedded with young residual soil. It is a concrete-gravity type dam, approximately 12 m high, in simple concrete, with a longitudinal section around 220 m in length, and spillway walls of reinforced concrete, forming a reservoir of around 700,000 m^3. It was constructed by subdividing the concrete-gravity monolith into 14 foundation blocks, separated by contraction joints, designated from Block 11 (located on the right abutment) to Block 1, and Block A3 to Block A1 (located on the left abutment). The dam was completed in 2012 and came into operation in 2013.

The dam was evaluated with focus on the influence of the geological structures on the geotechnical parameters in terms of stability and deformability, based on the marked discontinuities in the foundation. These features, although difficult to reproduce in anisotropic rock through conventional limit equilibrium methods, were satisfactorily simu-

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lated with the Finite Element Method (FEM), employing an anisotropic constitutive model.

Figure 1 represents the blocks directly influenced by the characteristics of the foundation. Blocks 10 and 11, not shown in the figure, are influenced by the characteristics of the abutment. Figure 2 shows the longitudinal section of the dam axis, corresponding to Fig. 1, and the dimension of the foundation with respect to the estimated rock surface. Block 4, the object of the present study, is highlighted and was chosen due to its representativeness of the foundation as a whole.

This paper is based on data available from geological mapping carried out across the full extent of the San Juan dam. The geotechnical parameters were obtained through drilling and laboratory tests for application of the Jointed Rock (JRM, Plaxis 2D) constitutive model. Therefore, the stress-strain behavior in a highly discontinuous and anisotropic medium was studied based on the characteristics of Block 4.

2. Materials and Methods

The methodology can be divided into three stages. The first consisted in the geological mapping of the area of the axis of the dam and its reservoir, highlighting the main units and geological structures. In the second stage, percussion and rotary drilling were defined and executed primarily in the region of the foundation blocks. In addition, undisturbed samples of residual soil were collected at the dam foundation in order to perform characterization and direct shear tests. The third stage consisted of the choice of Block 4 for the evaluation of the stress-strain behavior of the foundation through FEM modeling, based on the geological-geotechnical parameters obtained. The first Author has been directly involved in the three stages. The latter stage has also had the collaboration of the second and third Authors.

2.1. Geological-geotechnical parameters

From the preliminary geological-geotechnical mapping, rotary and percussion drilling have been scheduled in order to define the geological layers and representative geological-geotechnical cross sections. Out of the fourteen blocks that make up the San Juan dam, Block 4 was chosen because it represents the characteristics of the foundation in terms of geological materials and heterogeneities evidenced through subsurface investigations and field mapping.

Ten undisturbed soil samples were collected, eight in the San Juan Dam area and two in the Water Treatment Station, since the residual soils occurring in these locations are geologically similar.

To obtain the strength parameters of the residual soils, two series of direct shear tests were performed. The first series comprised samples molded with the test failure surface parallel to the foliation of the test specimen material (Fig. 3a). The second series comprised samples molded orthogonal to the foliation (Fig. 3b). The direct shear tests were performed on controlled deformation Wykeham Farrance machines, with automatic monitoring of the variables involved. The specimens were molded and placed in the shear boxes under a vertical stress close to zero, and the boxes were then inundated with distilled water.

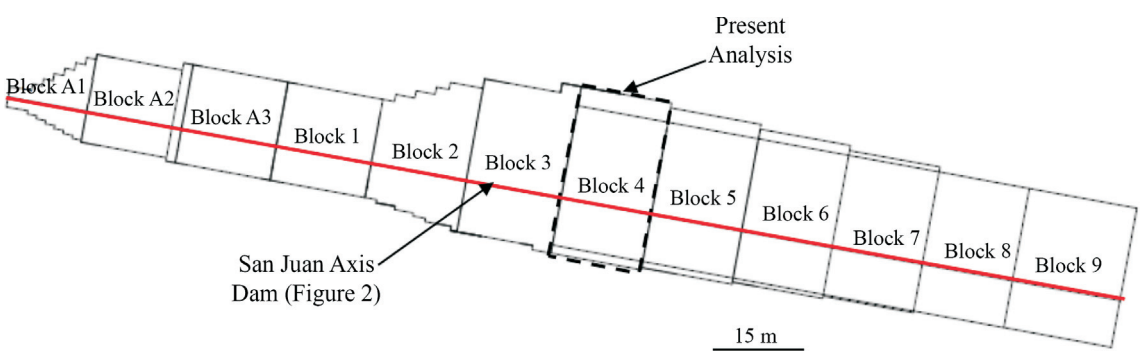

Figure 1 - General arrangement of the San Juan Dam. Original scale 1:250.

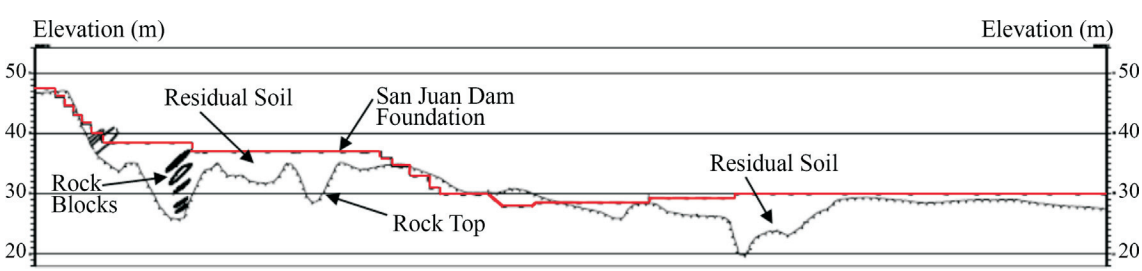

Figure 2 - San Juan Dam Axis Section (upstream) presenting the foundation elevations and the estimated bedrock surface. Original scale 1:250.

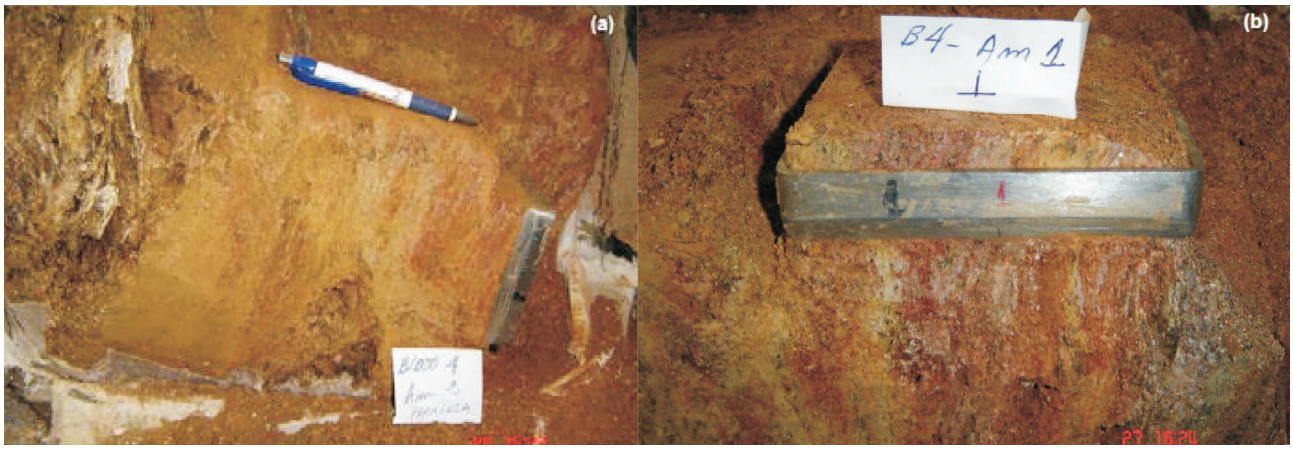

Figure 3 - Residual soil specimen in preparation for direct shear test: parallel (a) and orthogonal (b) to foliation.

The layers were grouped into three categories, according to the results of the N_{SPT} and direct shear tests, as follows: (i) F_q, for $N_{SPT} < 10$; (ii) F, for $10 < N_{SPT} < 30$; and (iii) R for $N_{SPT} > 30$. Because materials with penetration resistance $N_{SPT} > 30$ represent deformations deemed negligible for the elaborated study, the classification of the layers was simplified into three units for modeling. It should be noted that N_{SPT} refers to the number of blows for penetration of the final 30 cm of the sampler in percussion drilling according to Brazilian standards, which correlates with N_{60}, international standard, based on the relationship:

$$N_{60} = C \cdot N_{SPT} \quad (1)$$

The value of $C = 1.37$, in the above expression, instead of the value of 1.20 proposed by Décourt *et al.*, 1989, is an average value based on energy measurements performed in equipment routinely used in Brazil (*e.g.*, Belicanta, 1985, 1998; Cavalcante, 2002; Odebrecht, 2003).

2.2. Stress-strain behavior of the foundation

The geotechnical properties of the materials served as the basis for the modeling of the San Juan dam foundation, using the JRM constitutive model for the period immediately after the filling of the reservoir without seismic conditions.

The finite element method provides stress-strain analysis of continuous media representing complex behaviors and highly irregular geometries, diverse loading conditions, heterogeneities and non-linearity of materials. In addition, the JRM model takes into account the stratification and the particular directions of geological structures, such as foliations, faults and fractures. The JRM model assumes the regions between fractures as intact rock. The intact rock is linearly elastic, with transversely anisotropic behavior defined by constant Young moduli E_x and E_y and Poisson ratio ν. For the foliation and fracture planes, the shear stresses are limited according to the Mohr-Coulomb criterion. The application of the JRM model is limited to two families of joints or fractures, and is therefore a very useful and accurate model for stress-strain analysis of discontinuous rock masses (Xu *et al.*, 2015).

In this context, among the available sections for Block 4, the cross-section of the dam axis 4.1 was selected for modeling, given its representative characteristics and the geometry of the geological materials. A locally refined mesh (under the dam) was set, considering the irregularities of the foundation geometry, for soils and rocks, so that stress concentrations at these locations could be adequately modelled.

Two cases were considered for the modeling of section 4.1 (Fig. 4). In case 1 (left), the dam stability was simu-

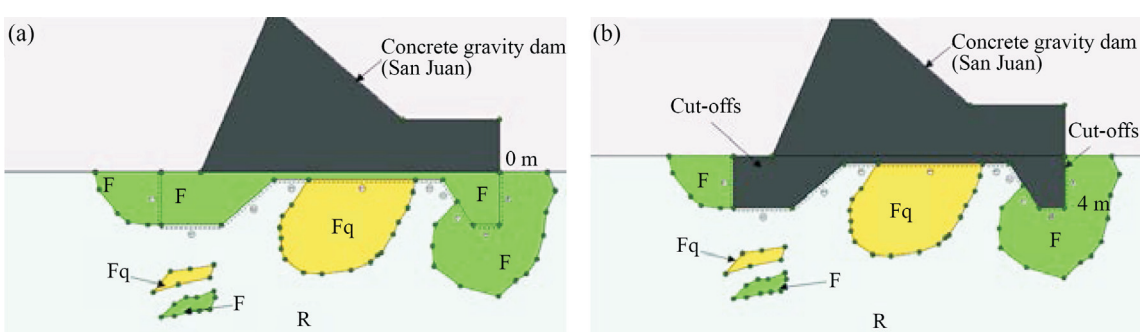

Figure 4 - Modeling the foundation of Block 4 at elevation 0 m. Case 1 (a): no cut-offs, and, Case 2 (b): with concrete cut-offs excavated to elevation -4 m.

lated with the foundation resting on a single elevation on young residual soil (el. 0 m), without reinforcement (cut-offs). Case 2 (right) simulated the foundation reinforcement with 4 m deep concrete cut-offs (base at el. - 4 m). For both cases, the water head totals 12 m upstream and 0 m downstream, representing the condition of dam operation. The dam is of an inclined bi-faceted type, that is, it forms acute and obtuse angles upstream and downstream, respectively.

The foundation was divided into three main units, based on their geotechnical properties, designated as F_q (very low resistance soil), F (low resistance soil), for young residual soils, and R (rock) for the sound rock mass. The dam body is represented by material C (concrete). The drained analyses assumed steady-state flow corresponding to the operating conditions of the dam.

3. Results and Discussion

3.1. Geological and geotechnical aspects

Geologically, the Samaná Peninsula is primarily composed of micaceous schist and marble with poorly defined boundaries (Mollat *et al.*, 2004), and occasionally occurring coverings of more recent limestone on top of the marble.

The San Juan dam is approximately 200 m downstream from a slightly narrower stretch of the river valley of the same name. Topographically, the left margin opens much more than the right, both covered by alluvial-colluvial sediments and residual soils. Basically, the geological units that occur in the studied area (Fig. 5), identified in field mappings, are: schist, marble and aluvial-colluvial soil. The materials that make up the foundation of the San Juan dam are young residual soils of schist, as well as less altered portions of this rock, intercalated, or not, with marble.

The area is usually devoid of rocky outcrops, including the abutments, except immediately downstream, on the right bank, where there was a massive outcrop of schist, one of the only ones at that location. Field observations did not reveal evidence of infiltration zones in the riverbed. The outcrops found at the margins indicate the presence of semi-altered or partially altered sheared schist, which produces a very clayey residual soil with low permeability.

The measurements of schistosity and fractures obtained for foundation Block 4 showed a preferential direction N75° W/25° SW~ N55° W/80° SW. Figure 6 shows, schematically, the principal directions of the discontinuities accounted for in the numerical model. These fracture families are parallel and make 30° and 60° angles (α) with a horizontal line, parallel to the dam foundation.

Figure 7 shows the diverse typical geological and geotechnical characteristics of the San Juan dam. In Fig. 7 (a), intercalations of schist (transitional soil / rock material) and marble with NW-SE foliation are presented in a section of the dam access road. Sometimes families of fractures are formed by the occurrence of karst cavities (Fig. 7b) in centimetric pores disseminated in the rock, and following rock foliation. The main fracture families of the rock mass are arranged in planes parallel to or normal to the foliation (Fig. 7c), and may be filled with carbonate and/or quartz material in the schist rocks.

Figure 8 shows the boundary of Block 4 in the San Juan dam, together with the geological-geotechnical sections (Fig. 9) used for the modeling process. In addition, the structural contour lines of the rock top are indicated with their respective elevations. All the geological units are re-

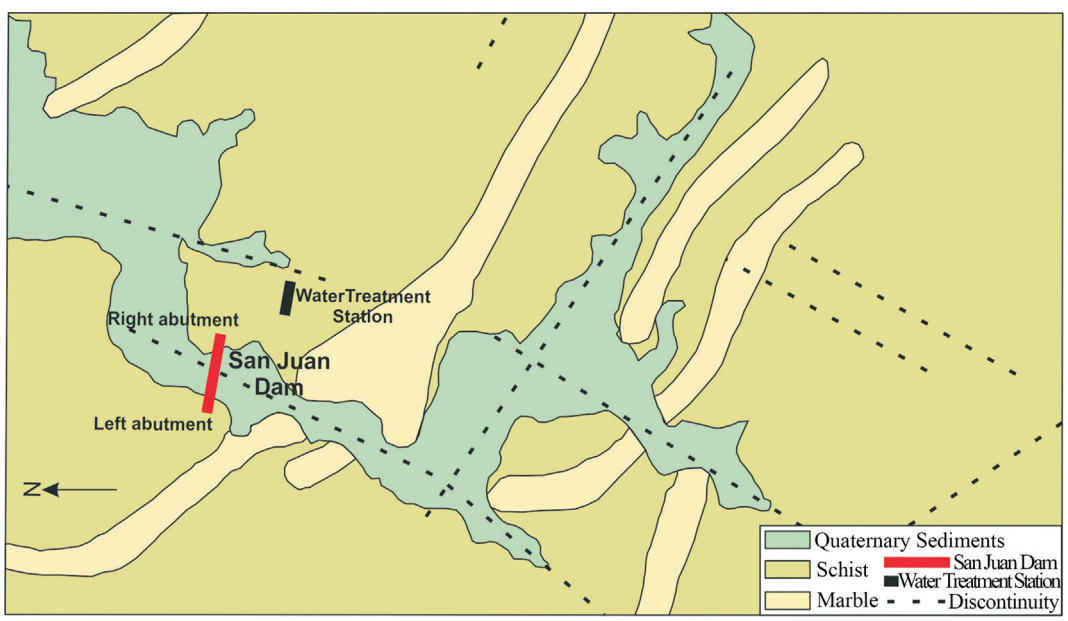

Figure 5 - Regional geological map of San Juan Dam Axis (in red) and Water Treatment Station (in black). Original scale 1:5.000.

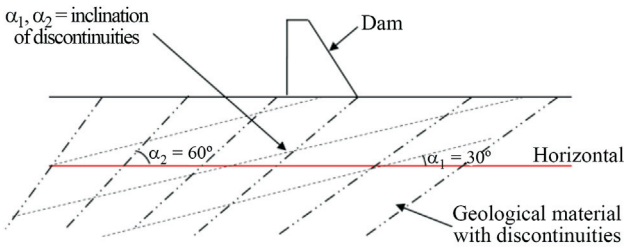

Figure 6 - Sketch showing the discontinuity family directions considered in the numerical model.

Figure 7 - Geological-geotechnical of geological materials.

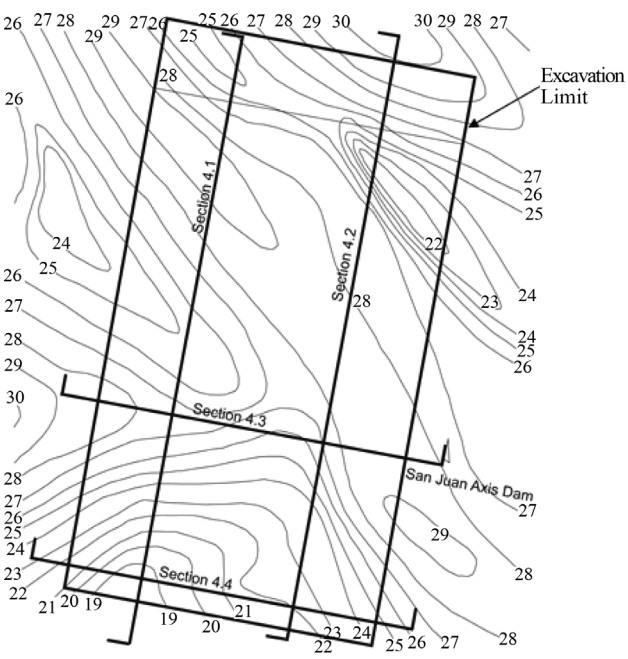

Figure 8 - Bedrock contours - Block 4, plan view.

sidual soils and altered schist rocks, intercalated with passages of marble.

In addition, in the young residual soil foundation, intercalations of weathered rock occur, coming from portions that are more resistant to weathering. Stretches of low resistance (N_{SPT} = 2) residual micaceous soil were identified in portions underlying the dam foundation.

Figure 10 shows sample boxes from a drilling survey in the region of Block 4, where passages of residual soil are noted below long rocky stretches. Figures 10 (a) and 10 (b) show intercalations of passages of mature and young residual soil and small stretches of heavily altered rock. In Fig. 10 (c) to Fig. 10 (e) it is possible to identify stretches of residual soil below extensive rocky stretches, altered or not (depths 10.45 m, 11.0 m and 18.10 m). In Fig. 10 (f), a transitional soil/rock passage (90% residual soil of schist and 10% schist rock) is evident in detail. Similar passages can be observed at points indicated by the arrows.

The results obtained from direct shear tests, both orthogonal (blue) and parallel (red), to the foliation are presented in Fig. 11. Since the foundation is very heterogeneous, composed of young residual soils of schist and heavily altered rock, a linear regression analysis was performed based on the Brazilian Standard NBR 11682 (ABNT, 2009) to obtain the effective parameters of c' and ϕ'. Confidence limits of individual strength (longer dashed lines) and mean resistance (shorter dashed lines) are included, with a 5% level of significance (two-tailed). Although the orthogonal results presented higher values than those parallel to the foliation, due to the influence of the discontinuities in the latter condition, a mean cohesion c' = 48 kPa and friction angle (ϕ') = 27 ° was adopted for the foundation in altered rock (R).

For the other materials (F_q and F), the values of c' and ϕ', as well as the other mechanical parameters required for modeling, are presented in Table 1. The permeability values included in Table 1 have been chosen in the middle range of those related to the effective stresses occurring in the dam foundation during the filling of the reservoir.

For selection of the geomechanical parameters of the geological materials that make up the foundation, according to Table 1, the filler materials between discontinuities were considered to represent the three units (F_q, F and R). According to the JRM model, the values E_2 and G_2 for the filler materials control, respectively, the normal and the shear stiffness along the discontinuities and therefore are mutually independent. Considering that the main objective of this work was to study the influence of the anisotropy on the stability of the dam, with no primary concern on displacements, the moduli in Table 1 have been selected arbitrarily, according to typical values representing the respective materials. Since the marked foliation in these units controls the stability, the selected effective strength parameters c_1' and c_2' for each unity were those obtained from the direct shear tests. For all natural materials, a tension cut-off of 100 kN/m^2 was assumed to control occasional tensile stresses.

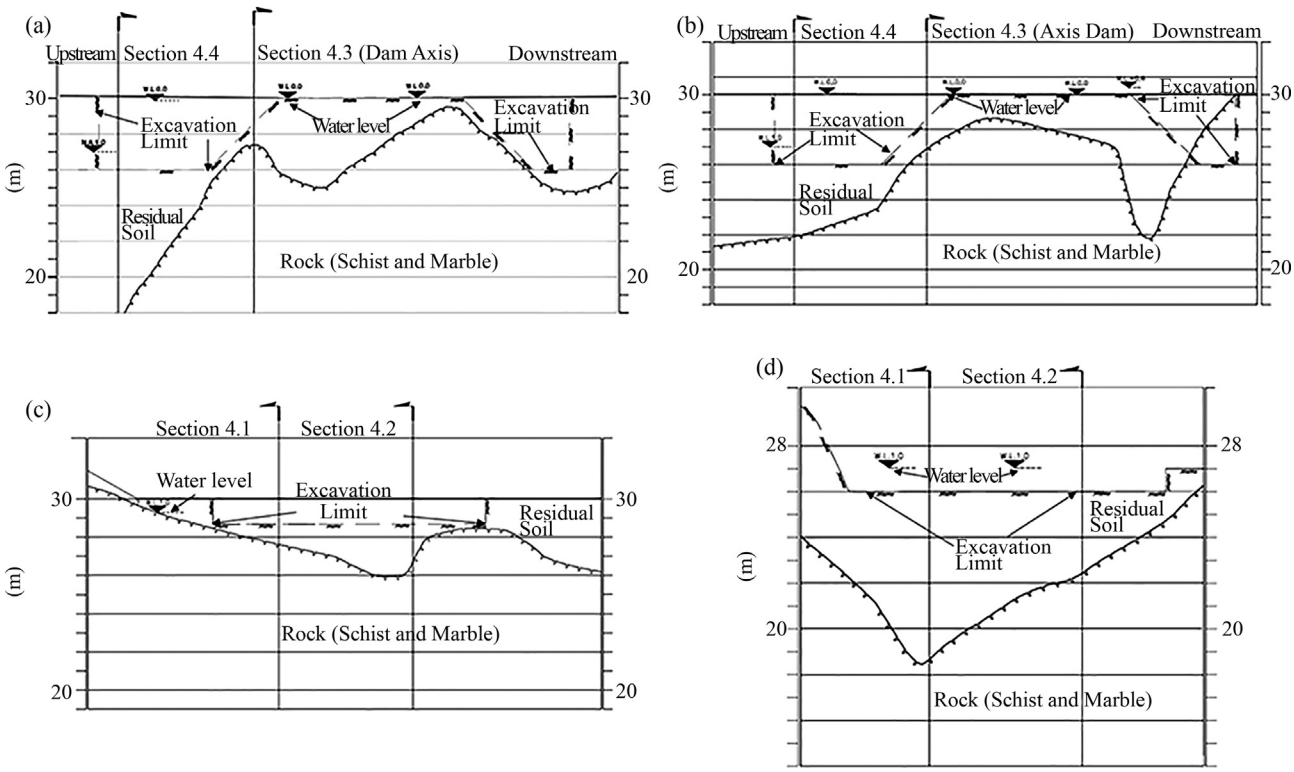

Figure 9 - Sections 4.1 (a - study object), 4.2 (b), 4.3 (c) and 4.4 (d) of Block 4. The sections show the boundary between the estimated rocky top (altered rock) and the young residual soil of the foundation. The excavation dimensions of the foundation and the water level obtained by drilling are presented.

Figure 10 - Drill core samples (a-e), in sequence, from SM-19 (rotary drilling) in Block 4 dam foundation axis. In detail (f) transitional material (soil/rock), depth 10.45 to 11.0 m.

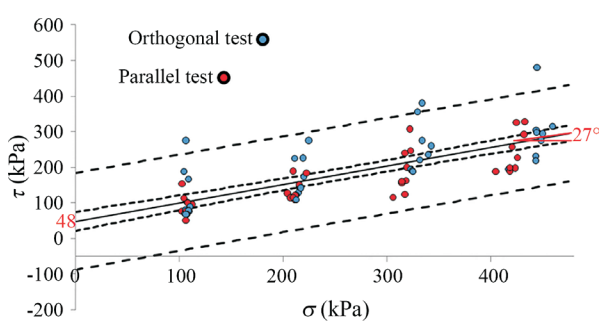

Figure 11 - Confidence limit intervals, 5% significance.

3.2. Flow, stress and strain analysis

The directions of the flow vectors are presented sequentially for Case 1 (Fig. 12a) and Case 2 (Fig. 12b).

For Cases 1 and 2, there was not much difference on the maximum pore-pressures, of 91 to 112 kN / m^2, respectively. Likewise, in both cases, the flow velocities presented low values, of the order of 0.005 and 0.007 m / day, due to essentially clayey soil. However, due to the head loss provided by the cut-offs, caused by the longer flow path, the output gradient is clearly smaller in Case 2. The cut-offs

Table 1 - Geological-geomechanical parameters for the FEM analysis.

Materials	γ_{sat} (kN/m^3)	E_1 (kN/m^2)	ν_1	Modeling parameters - jointed rock model									
				E_2 (kN/m^2)	ν_2	G_2 (kN/m^2)	c_1' (kN/m^2)	c_2' (kN/m^2)	ϕ' (°)	k (cm/s)	α_1 (°)	α_2 (°)	Tension cut-offs (kN/m^2)
F_q	18.6	9.5x10^3	0.2	3.0x10^3	0.2	1.5x10^3	10	10	0	2.9x10^2	30	60	100
F	20.0	2.0x10^4	0.2	3.0x10^3	0.2	1.5x10^3	20	20	27	1.1x10^2	30	60	100
R	22.0	3.5x10^4	0.2	3.0x10^3	0.2	1.5x10^3	48	20	27	0.5 x10^3	30	60	100
C	25.0	2.5x10^7	0.2	-	-	-	-	-	-	-	-	-	-

Legend: F_q (very low resistance soil); F (low resistance soil); R (weathering rock) and C (concrete). γ_{sat} (saturated specific weight) E_1 (Young modulus for the intact rock), ν_1 (Poisson ratio for the intact rock), E_2 (Young modulus for the fill materials), ν_2 (Poisson ratio for the fill material), G_2 (shear modulus of the fill materials), c_1' and c_2' (cohesion of the fill materials), ϕ' (friction angle), k (permeability coefficient, obtained by Lefranc and Lugeon tests), α_1 and α_2 (angles of the discontinuities for planes 1 and 2, respectively, with the horizontal).

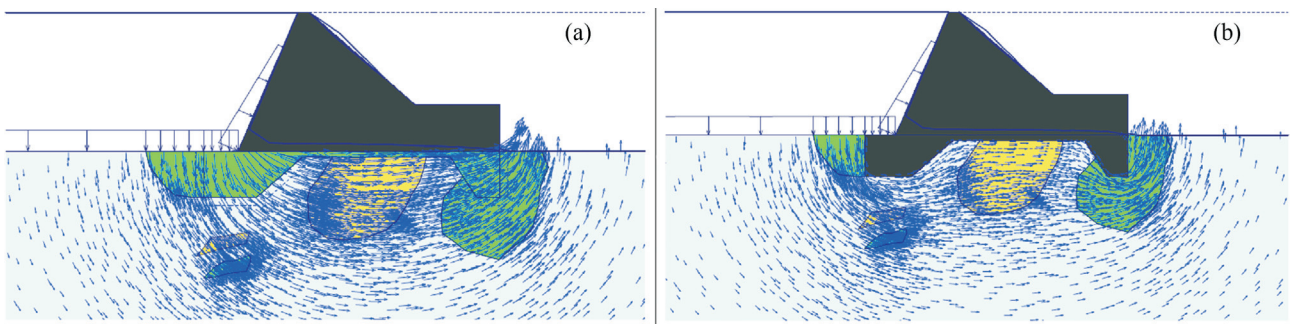

Figure 12 - Flow vectors generated for the foundation conditions for Case 1 (a) and Case 2 (b).

clearly increased the strength of the foundation mass, increasing the load capacity of the dam to withstand stress.

Figure 13 shows the total displacement contours corresponding to the deformation moduli assumed. Regardless of the values assumed for the material moduli, the highest total displacements were concentrated in the upstream toe, progressing downstream in a direction nearly perpendicular to the α_2 - inclined discontinuities. This indicates that there are no significant displacements along the discontinuities, ensuring the adequate stability of the dam. The maximum displacements found for both Cases 1 and 2 were close, around 14 mm for the F and F_q soils below the dam foundation. In terms of specific deformations, they were larger for Case 1 (5.93%) than Case 2 (2.83%), with respect to the F_q layer. This behavior was expected, since the execution of the cut-offs removed a large portion of the deformable layers of low resistance soils (F_q), up to 4 m thick, replacing them with concrete.

The plastic points (Fig. 14), which represent points in the soil that reached the Mohr-Coulomb envelope, are mainly concentrated in the F_q and F materials, primarily near the foundation or cut-offs of the dam. In Case 2, since a portion of the materials (F and F_q) were excavated and replaced with concrete, the plastic points tended to move farther away from the dam-foundation contact (Fig. 14b).

The safety factors for Cases 1 and 2 were, respectively, 1.82 and 2.61, obtained numerically by the progressive reduction of the parameters c' and ϕ', proving that the excavation up to the depth -4 m significantly increased the stability of the dam with a foundation of predominantly

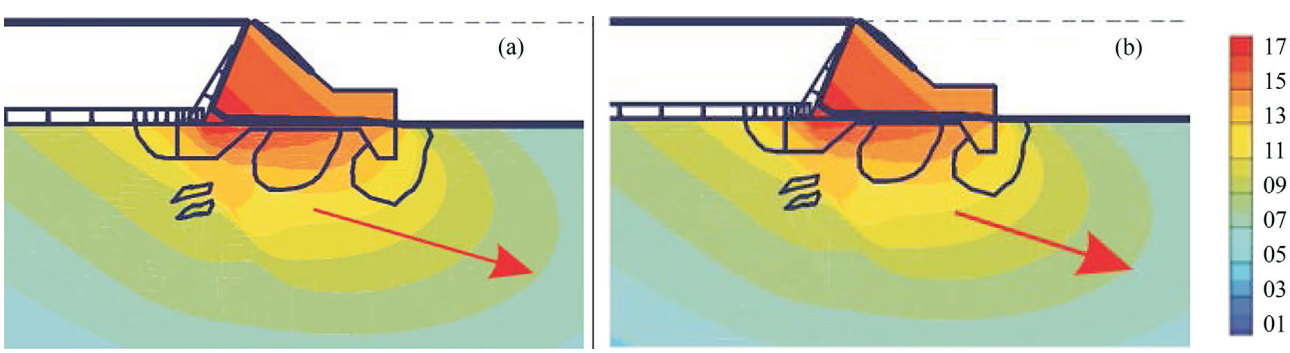

Figure 13 - Displacement contours (cm) generated for the foundation conditions for Case 1 - (a) and Case 2 - (b). The arrow in red highlights the predominant deformation direction and the color scale represents the displacements (mm).

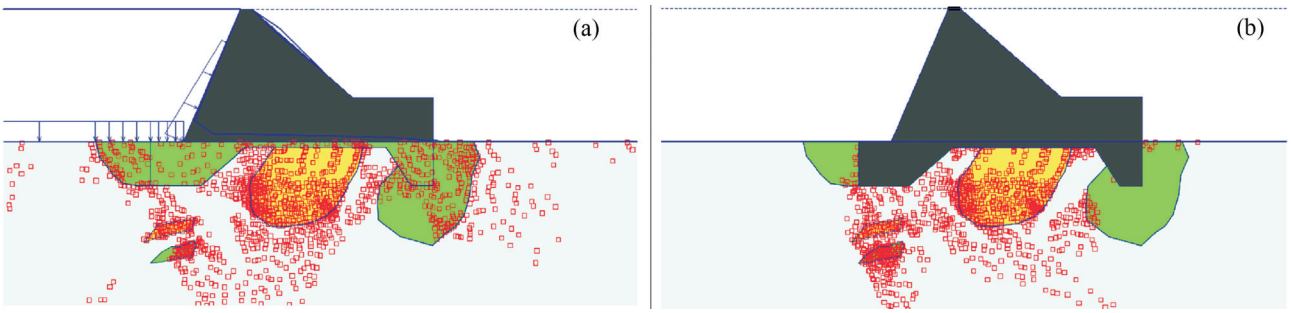

Figure 14 - Plastic points (red) obtained for Case 1 (a) and Case 2 (b).

anisotropic soil. The JRM model was therefore demonstrated to be suitable for the stress-strain analysis in question.

4. Conclusions

The methodology used for the development of this article addressed the geological and geotechnical peculiarities of the foundation material of the San Juan Dam, taking into account both its great heterogeneity and anisotropy, as well as the constitutive properties of the soils and rocks existing in the region. Materials with these characteristics are often discarded as foundation options. However, in places where weathering acts in the development of extensive alteration profiles, the understanding of the geomechanical behavior of these materials may favor the construction of more daring projects: in the case of dams, the construction of concrete dams rather than earth dams.

The strong anisotropy of the foundation materials was determinant in the choice of the constitutive model type (JRM), valid for investigating stratified layers with two dominant directions. The results obtained through FEM modeling revealed that this type of analysis is very useful in the design stages, guiding actions during construction, as well as allowing for bolder solutions for the San Juan Dam.

Acknowledgments

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